Remedial Project for Kölnbrein Arch Dam
Design and Construction

Österreichische Draukraftwerke AG.
Carinthia/Austria
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The unexpected problems that occurred at the Kölnbrein dam of the Malta power system during the third storage period in 1978 were a great challenge to dam experts from at home and abroad. A long distance was covered from the initial supplementary precautions to the finding out of the cause of the cracks and up to the final remedial project, which was started in 1989.

This distance, competently accompanied by the internationally acknowledged dam expert Dr. Giovanni Lombardi, was for that reason long and doubtlessly difficult, due to the fact that world-wide no similar problem with the same kind of dam behaviour was known. The dam experts for the remedial works broke new ground, which before then had never been trodden by anybody. A great deal of painstaking detailed work has finally led to a project which is not only unique but also very promising.

In the publication year of this Brochure, namely 1991, we are right in the middle of the remedial works, which consist of structural, mechanical and electrical sections.

The 17th ICOLD-Congress in Vienna gives Österreichische Draukraftwerke AG as the owner and operator of the Malta power system and the companies engaged in the remedial works the chance to demonstrate the exceptional and expensive remedial process by means of this Brochure. We think that this is an important contribution towards a better understanding of the exceptional problems.

It should be pointed out, however, that naturally the articles contained in this Brochure should be regarded as interim reports.

Our thanks go to the authors and staff members responsible for the realisation of this Brochure as well as to the assisting companies.

ÖSTERREICHISCHE DRAUKRAFTWERKE
AKTIENGESELLSCHAFT
Board of Directors
The Malta hydro power complex is situated on the southern slope of the Hohe Tauern mountains in the province of Carinthia and comprises in fact three power schemes — two storage schemes and a downstream run-of-river station which utilises both the discharge from the upstream reservoirs and the flow of the River Möll. The main feature of the development is the Köllnstein seasonal reservoir with its live storage capacity of 200 million m³ of water corresponding to an energy equivalent of 588 million kWh. Fig. 1/1 below is a summary of the relevant operational data.

The development was constructed between 1971 and 1978. The magnitude and suprarregional importance of the hydro potential available in the Malta, Lieser and Möll river basins led the Austrian government, under the 2nd Nationalisation Act, to entrust power utility Österreichische Drau-Kraftwerke with the planning, design, construction and operation of this power project. First power was generated by the development as early as December 1976, with partial reservoir filling while construction was still underway. Full operation was commenced in 1979.

From the beginning of partial operation to the end of 1990, thehydro power complex generated a total of 11,300 million kWh with a winter proportion of 60 percent. This corresponds to some 2 percent of Austria's total domestic requirements, as compared with the approximately 5 percent supplied by the largest power station on the Danube at Attenworth. The importance of the storage scheme, however, is not solely a function of the magnitude of its annual generation. Its essential merit is that a large capacity range is available more or less independently of the momentary water yields. This availability is not afforded by pure run-of-river stations. It is in fact only by the combined operation of storage and run-of-river schemes that the utilisation of hydro power as an environmentally compatible and renewable source of energy can be adjusted to the requirements of electricity supply.

The Malta development has an installed (maximum) turbine capacity of 291,000 kW and a pump capacity of 406,000 kW, so as to represent a capacity range of 1,297,000 kW. With a maximum capacity of 730,000 kW, the Malta Main Scheme is Austria's most powerful single hydro station. The power units of the storage schemes — four Pelton turbines, of which two are turbines only and two are pumps-turbines, in the Main Station and two Isogyr pumps-turbines in the Upper Station — have the shortest possible run-up times, allowing full load to be reached within a few minutes. Change-over from generation to pumping and vice versa is also possible within a few minutes.

Data in Million m³ and (GWh)

![Diagram](image)

**Fig. 1/1: Main Data**

This flexibility in operation, combined with an average annual generation of as much as 905 million kWh, makes the Malta development ideally suited to fulfill special tasks, as peaking, system control, stand-by, and energy upgrading. For system control operation, the power units of the storage schemes are run in accordance with the requirements of the system. The constant changes in loads involved in this mode of operation make high demands on the hydraulic system. Apart from its function of system control, the development is also run according to a fixed schedule or to meet instantaneous requirements. Thanks to its large capacity and ready availability, Malta may step in when power station failure occurs, not only in Austria but also across the national boundaries. Furthermore, the development is integrated into a scheme of an automatic sequence of emergency measures designed to lessen the effects of major faults.

Besides the electrical characteristics as e.g. voltage and frequency, flexibility is an essential quality requirement in electricity supply. The consumer's right to meet his electricity requirements from the wall socket at any time may be at variance with the natural pattern of water yields, especially in a country that is dominated by hydro power. Power generation in Austria originates from run-of-river stations for more than 50 percent, so as to be dependent on precipitation patterns, which are subject to substantial seasonal variations. Storage reservoirs capable of retaining large amounts of water may balance these fluctuations.

Reservoirs may be filled both by damming up natural streamflow and by pumping water from lower levels using electricity from surplus generation in run-of-river stations. This so-called pumped-storage operation serves to upgrade energy by producing valuable peak energy from surplus energy. The Main Reservoir created by the Köllnstein dam is designed so as to be filled for about 45 percent of its capacity from natural inflow. The balance is provided mainly by pumping from the level of the Main Station. In addition, provisions have been made to permit the pumping of water from the River Möll against a head of more than 1,000 m. For this purpose, the Wörth power plant has been equipped with pumps with unit capacities of 2 times 80,000 kW and 2 times 145,000 kW. This allows operation to be adjusted to the variable conditions of electricity consumption.

Apart from storing summer run-off for use during the winter months, provisions have been made for a daily pumping cycle, where water is pumped up during the night hours for use at times of peak load during the day. This may increase the scheme's annual generation by as much as 440 million kWh. The daily pumping cycle is particularly useful when combined with the operation of large thermal power stations.
so as to benefit the nation’s economy as a whole. Retention of flood peaks reduces flood risks and resulting damage for downstream reaches. The peak flows involved may be used for meeting peak loads in consumption rather than flowing unused over the spillways of downstream power stations on the River Drau. The additional generation involved corresponds to more than 30 million kWh p.a. Finally, the storage afforded by Kölnbrein may assume importance as a drinking water reserve for the future.

**Repair of Kölnbrein Dam**

In the years 1977 to 1984, the Kölnbrein reservoir was operated in accordance with the requirements of the system. The designed reservoir top at EL. 1902 m a.s.l. was reached twice during that period. However, when cracking in the base led to increased leakage, authorities reduced the permissible water level in the reservoir to EL. 1880 m a.s.l. This involved an about 26 percent cut in live storage and curtailed the possibility of saving summer runoff for winter generation. This is clearly demonstrated by Fig. 1/2 showing aggregate generation, specified by summer and winter terms, since the development was placed in service. Due to the restriction on reservoir filling, winter generation has suffered an average reduction of 20 percent for the benefit of summer generation. Annual generation is little affected by the restriction on impounding. Further restrictions have been imposed as remedial works proceeded on the dam.

Storage capacity is a valuable asset and will increasingly be so in the future, as the number of sites with topographies suitable for installing new reservoirs is limited. In addition, there is a growing resistance on the part of the public to hydro power projects at high level locations in the Alps, which in fact offer the combined advantages of sufficient water yields and large heads.

Failing efficient rehabilitation of Kölnbrein dam, further reductions in live storage would have to be faced in the course of time.

In the light of what has been said above, both concerns over the ways of dealing with landscape and considerations of power economy have been arguments in favour of undertaking the repair of the dam.

The Malta development is Austria’s largest and most powerful storage complex. It has given proof of its efficiency and has made an essential contribution to the Austrian electricity supply ever since it was built.

**Main Data**

The total average annual energy from the Malta power system is 905.6 million kWh, of which 712.8 million kWh, or 78.7%, is generated during the winter half-year.

Assuming maximum possible energy generation from daily pumping in the Malta Main Stage, corresponding to 440 million kWh, the maximum possible generation is brought to 1365 million kWh p.a. on average.

<table>
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<th>Reservoir capacity 1)</th>
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<td>(Upper Stage + Main Stage) Gross head</td>
<td>m</td>
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<td>Rated capacity Turbines</td>
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<td>Pumps</td>
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<tr>
<td>Pumping energy consumption for reservoir filling</td>
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<td>193.0</td>
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</table>

**Notes:**

1) Not including net capacity of surge space in Malta Lower Stage
2) Relative to Malta Main Station
### TABLE SHOWING MAIN DATA

<table>
<thead>
<tr>
<th></th>
<th>Upper Stage</th>
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<td>-</td>
</tr>
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<tr>
<td>Discharge</td>
<td>M m³</td>
<td>208.2</td>
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<td>M m³</td>
<td>208.2</td>
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</table>

**Reservoir capacity**
- M³

**Energy stored in reservoirs**
- M³

**Mean specific energy**
- kWh/m³

**Gross head**
- m

**Rated discharge**
- m³/s

**Rated capacity**
- kW

**Generation**
- M kWh

**Pump energy absorbed**
- M kWh

### Notes:

1) Water from inactive storage (below minimum operating level) in Kölntrein reservoir can be used as compensation water released through the bottom outlet and conveyed to the Galgenbock basin.

2) Related to the centre of gravity of Kölntrein reservoir.

3) Inflow to Rottau compensating basin not including power water from Malta Main Stage.

4) Surge capacity in Lower Stage.

5) For power unit of weir, 0.03.

6) Compensation water discharge through weir turbine and into the river Möll.

7) of which 4.8 million kwh in annual energy from weir generator.

### Diagram

**Gesamt-erzeugung** - Total energy generation

**Verluste an Wintererzeugung durch Stauziel-begrenzung** - Losses from limited winter generation

**Winter-erzeugung** - Winter generation

**Sommer-erzeugung** - Summer generation

**Teilbetrieb** - Partial operation

**Vollbetrieb** - Full operation

**Stauziel-begrenzung** - Storage capacity limit

*Fig. 1/2: Power Generation from the Malta development between 1976 and 1990*
The Kölnbrein arch dam, completed in 1977, creates a reservoir with a live storage capacity of 200 million m³ of water. Being 626 m long at the crest and 1.6 million m³ in concrete volume, Kölnbrein dam is among Europe’s highest arch dams and, having to carry a water load of 54 MN (5,400,000 t), ranks high even among the world’s arch dams. The dam and reservoir are situated entirely within the Central Gneiss formation of the Ankogel-Hochalm group in the eastern Hohe Tauern mountains.

2.1 Dam Monitoring
From the very beginning, the Kölnbrein dam was provided with an extensive surveillance system which allowed detailed comparison with the analytical results (Fig. 2/1). From 1976, about 400 instrument locations were available, out of which about 300 were connected to a telemonitoring system with automatic data recording. Data from this monitoring system was submitted to prove that during the first two filling periods in 1976 and 1977 the Kölnbrein dam with the reservoir water surface at El. 1812 and 1854, respectively, had performed as expected in the design.

2.2 Seepage Loss and Uplift Pressure
During reservoir filling in the autumn of 1987, as the water level passed El. 1960 and continued to rise to El. 1981 m, i.e. 11 m within top water level, the following observations were made (Fig. 2/2):
- Seepage flows emerging from the drains in the lowest inspection gallery increased substantially and reached values of more than 200 l/s;
- uplift and joint water pressures at the base of the highest blocks reached values corresponding to a maximum 100 percent of the head of water behind the dam.

The conclusion drawn from these observations was that a tension zone must have developed in the upstream base of the highest blocks, breaching the grout curtain, which extends a maximum 70 m below the dam.
2.3 Supplementary Structural Measures between 1979 and 1985

In order to reduce objectionable uplift pressures and seepage losses and to answer the need for maintaining an at least restricted level of power station operation, the following remedial works were undertaken in the years that followed (Fig. 2/3):

- In 1979, the grout curtain sunk from the lowest inspection gallery was reinforced by supplementary grouting. Additional drainage holes were provided to relieve uplift pressure.

- In 1980 and 1981, artificial resins (PU resins) were pumped into the upstream tension zones of some of the centre blocks, while in the remaining
blocks a freeze curtain about 4000 m² in area and with a maximum thickness of 5 m was provided as a temporary measure using the method of artificial foundation freezing. Water was frozen during reservoir filling and thawed during emptying.

- Between 1981 and 1983, a reinforced-concrete slab about 4000 m² in area and varying between 2 and 10 m in thickness was provided as an upstream apron connected to the upstream dam surface by an elastic joint. The aim of this measure was to remove the impervious element from the overstressed area and relocate it in the intact reservoir floor.
- In 1984 and 1985, additional plastic foil had to be provided at the surface of the apron, and foundation grouting had to be repeated. These measures, taken in the months of February to May of the respective years, made it possible for the Kölnbrein reservoir to be filled to more than 90 percent up to 1984 and twice to maximum operating level, in 1979 and 1983 (Fig. 2/4).

2.4 Exploration of the Crack Zones

Simultaneously with the above supplementary measures, more than 33 000 linear metres of percussion and core drillings were sunk in an effort to obtain detailed information on the crack zones in the dam base. The cracks so established tend to concentrate in two main zones (Fig. 2/4):

- Downstream crack zone
  Cracks in the foundation rock, running approximately parallel with the dam base, and cracks following horizontal construction joints in the dam concrete, extending to a maximum 11 m into the dam. Maximum crack opening with an empty reservoir is 3.5 mm.

- Upstream crack zone
  Shell-shaped cracks, rising steeply to, and daylighting at, the upstream face of the dam, both in the concrete and in the foundation rock. These cracks must have formed in the upstream one-third of the base of Blocks 12 to 21 whenever the water level in the reservoir rose beyond approximately El. 1689 m, during the reservoir filling periods of the years 1978 to 1983. The cracks have been water-filled to the present day and reached openings of up to 30 mm when the reservoir was filled to top water level for the first time in 1979.

2.5 The Causes of Cracking

The cracks in the downstream dam base are believed to have developed towards the end of the construction period as a result of vertical tensile stresses from dead load, and possibly to have widened following joint grouting. At first it was suspected that unexpectedly high vertical tensile stresses at the upstream dam toe were responsible for the cracking in the upstream dam base. In fact, this phenomenon is known to be a relatively common cause of near-horizontal cracks at the upstream base in arch dams, either in the concrete above, or in the rock foundation below. Potential causes of such increased tensile stresses could have been e.g.:

- settlement of the reservoir floor not allowed for in the design,
- stage-wise contraction joint grouting for partial reservoir filling during the construction period and the resulting reduced compressive stress from dead load at the base of the blocks, which are constructed as statically independent monoliths,
- too pessimistic an assumption for the ratio of deformation moduli between concrete and rock of less than unity underlying the design, leading to increased loading from water load at the base section.

Thorough study of the crack pattern revealed, however, that the upstream cracks must all have started from the foundation contact and have continued along a sloping path towards the upstream so as to daylight at an acute angle at the upstream surface of the dam. G. Lombardi pointed out for the first time in 1985 that the crack pattern suggested the shear forces as potential causes of the development of sloping main tensile stresses in the dam base.

Later studies using failure tests conducted in pursuance of the above idea have shown that:

- under certain conditions the high shear forces lead to main tensile stresses being larger in the interior of the dam section than at the surfaces,
- residual tensile stresses from the hardening of the concrete may have intensified this tensile stress field,
- the sum total of these tensile stresses,
- which rapidly decline with increasing distance from the base led to initial cracks allowing water to penetrate, which then widened the cracks.

Fig. 2/5: Crack pattern in the dam foundation contact
3. Development of the Remedial Project for Kölnbrein Dam

The development of the sloping cracks, which is believed to have reached its final stage by the time the reservoir level had risen to El. 1860 m, converted the base of the arch dam into an articulated joint, which precludes the possibility of high vertical tensile stresses having built up at the upstream dam toe. Cracking has certainly relieved the residual tensile stresses from the construction period as well as water pressure in the cracks resulting from the reduction of uplift pressure. The remedial project aims at halving shear stresses by means of a downstream thrust block intended to take up horizontal stresses below El. 1760 m.

As the supplementary measures described under 2.3 above did not lead to any satisfactory improvement in dam behaviour, the High Water Right Authority, in 1984, reduced the permissible water level in the reservoir from 17 m and finally to 22 m below the designed maximum operating level 1902 m. This led Österreichische Dauerkraftwerke to entrust Dr. Lombard with the preparation of a remedial project with the following objectives:
- Increasing dam stability within the crack zone and
- Ensuring lasting unrestricted reservoir operation at Kölnbrein.

3.1 Project Idea

The fairly thin Kölnbrein arch dam spans a wide and flat-bottomed valley. Arch dams built at valley sections of this configuration are known to develop very high stresses within the cantilever sections at the base. In addition, at Kölnbrein near-horizontal cracks have developed in the lower downstream portion of the dam, which have substantially weakened the respective sections.

During reservoir filling, redistribution of forces leads to a reduction in vertical weight component in the centre portion of the dam, while naturally the downstream directed transverse forces steadily increases in the lower part of the dam, so that, as the water level rises in the reservoir, the ratio of transverse force to normal force steadily deteriorates. This phenomenon finally caused the centre blocks of the Kölnbrein dam to shear off near the base, with the planes of failure forming a complex pattern mainly consisting of the above mentioned near-horizontal cracks on the downstream side and steeply sloping cracks upstream.

Various attempts at remedial action made in the course of the years have demonstrated that grouting or similar measures are not sufficient to ensure the lasting safety of the overstressed sections in the centre portion of the dam base. Therefore, efficient remedial action must aim at either increasing the vertical normal force substantially or reducing the transverse force correspondingly in the critical zone of the dam, in such a way that the ratio between the two forces does not exceed a certain limit. In addition, provisions should be made to ensure that the range of variation of the moments between the reservoir-empty condition and the reservoir-full condition be restricted so that the resultant remains within the core zone of the base section.

In order to satisfy these conditions, various possibilities were developed and studied in detail, which all aimed at increasing the vertical force or reducing the transverse force, or at a combination of the two. Thorough study and consideration of the advantages and disadvantages of the various proposals finally led to the selection of a project providing for a heavy downstream arch-gravity structure as a thrust block propping the dam. The structure would take up a share of the load in the order of 1.2 million t and at the same time would reduce the transverse forces in a corresponding amount. The thrust block is to afford any substantial resistance. Its connection with the dam must not be too rigid, as otherwise the problems arising at its upper edge would be similar to those the new structure is intended to solve at the dam base (Fig. 3.1/1).

A solution was found for what appeared to be the conflicting requirements of stability and rigidity by providing a joint or gap with a variable opening between dam and supporting
structure. During reservoir filling, the gap closes progressively from the bottom upwards in accordance with characteristic curves to be calculated in advance and, if necessary, to be adjusted during the first phases of reservoir filling (Fig. 3.1/2).

Neoprene-pad bearings especially designed for this purpose will be provided to ensure that only those forces which are normal to the arched dam surface are transferred between dam and thrust block, so as to exclude shear forces both in the horizontal and vertical directions.

This type of pad has been preferred over other alternatives, as on the one hand the bearings are easily adjusted in the unloaded condition with a low reservoir level and, hence, an open gap and, on the other hand, when contact pressure between the two structures has been established, cannot be changed, neither due to faulty manoeuvring or human and mechanical error, nor maliciously.

The exact determination of the point of closure for each of the more than 800 bearings, that is, the determination of the reservoir levels at which the bearings get into contact with the dam and are intended to start assuming loads, constitutes a main element of the project idea. Another advantage of the massive thrust block adopted lies in the fact that the very weight of the structure substantially improves stress and stability conditions in the rock foundation area downstream of the dam.

Apart from the supporting structure, the remedial project provides for the repair of the cracks by grouting. Actually, the grouting scheme is one of the bases underlying the project and certainly also the stability analysis. The grouting is intended to ensure continuity in terms of transfer of forces and watertightness in the crack zones both of the foundation rock and the concrete.

The thrust block and in particular the magnitude of bearing pressure from the dam to be carried by it have been

Fig. 3.1/2: Working design

Downstream thrust block in the winter of 1990-91
dimensioned in such a way that the stresses caused in the lower portion of the concrete structure are limited to a narrow and optimal range during the reservoir filling and drawdown cycles. Primarily, however, provisions have been made to ensure that such cycles may take place without causing any objectionable tensile stresses.

A fact that is not to be overlooked is, however, that the final state of stress will be dependent on the manner in which the grouting is performed. The sequence and timing of the grouting operations as well as the procedures and pressures applied and the amounts of grout pumped in will be important factors influencing the success of the scheme.

The grouting operations will be carried out in seven phases termed "M to R". A drainage system will be provided in a phase "T" (Fig. 3.1/3).

A hyperbolic relationship between grout take and permeable pressure has been imposed for cement grouting in rock. The maximum allowable pressure will constantly be reduced as a function of the amount of grout already absorbed, with certain limits having to be observed in addition for pressure and grout take (Fig. 3.1/4). For synthetic-resin injections in concrete, it is primarily the volumes to be pumped in that have been determined as a function of crack width and desired reach. The reach is a function of the distance from the nearest borehole.

Dam monitoring during the grouting operations will concentrate on the local behaviour of the concrete mass in the vicinity of the injections as well as on the overall behaviour of the dam structure. No need to emphasise that appropriate instrumentation will have to provide for an accurate monitoring of the grouting operations. The existing dam instrumentation will be supplemented for this purpose. The main objective of this is to observe the widening of the cracks during the grouting operations so as to prevent their excessive opening.

On the basis of the above idea, the project was refined in a number of phases and finalised for execution. It is obvious that the uniqueness of the damage caused and of the problem to be solved has led to a novel solution. Still, it should be stressed that all the structural elements to be used are well tested so that no unforeseen events are expected to occur during construction or operation.

![Fig. 3.1/3: Grouting phases](image)

The planned grouting schedule is shown in Table 2.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Designation</th>
<th>Grout</th>
<th>Reservoir water level during grouting</th>
<th>Purpose</th>
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<td>Rock beneath thrust block</td>
<td>Cement</td>
<td>any</td>
<td>Consolidation (performed in 1990)</td>
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<tr>
<td>N</td>
<td>Rock beneath dam and apron</td>
<td>Cement</td>
<td>middle</td>
<td>Consolidation (partly performed in 1990)</td>
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<td>O</td>
<td>Concrete and upstream wedge</td>
<td>Rodur</td>
<td>middle</td>
<td>Anchoring (postponed)</td>
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<td>P</td>
<td>Concrete Crack Zone 1</td>
<td>Rodur</td>
<td>middle</td>
<td>Consolidation Sealing (performed from 1991)</td>
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<td>middle</td>
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<tr>
<td>S</td>
<td>Rock (Curtain connecting to dam)</td>
<td>Rodur</td>
<td>middle</td>
<td>Sealing (deep grout curtain)</td>
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<td>T</td>
<td>Rock drainage downstream</td>
<td>Rodur</td>
<td>any, following completion of grouting</td>
<td>Uplift reduction</td>
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Fig. 3.1/4: Pressure-volume diagram

Kölnbrein Dam
3.2 Site Geology

The 200-m high Kölnbrein dam, a thin arch dam, is located in a glacial-carved U-shaped valley within a unit of granitic gneiss that has been eroded upwards in a geologically very recent past (Fig. 3.2/1), and so are the other features of the Malta group of power schemes, as the minor Galgenbichl and Gösskar dams as well as all the diversion tunnels and the greater part of the power tunnel. Only the last part of the headrace tunnel, the penstock and the Rottau power station come into contact with the metamorphic sedimentary rocks of the so-called Schiefenhölle, or "slate mantle". In the south, where the terrain descends to the Möll valley.

The core of largely massive granitic gneiss includes rests of schistose rocks which are not entirely melted when the granite solutions intruded. Such schistose intercalations also occur in the foundation for the Kölnbrein dam (Figs. 3.3/3 and 3.2/4) and have clearly affected its deformation pattern (Fig. 3.2/2).

The site for the Kölnbrein dam was selected as being the most suitable one in terms of geology among four narrow sections in the Malta valley. First drillings and detailed geological mapping were undertaken as early as 1957. In the years that followed, geological investigations continued, including core drilling to a depth of 80 m. In situ radial jack tests were carried out in two out of the three exploratory galleries driven. In addition, a great number of rock mechanics tests were performed on specimens and drill cores.

Seismic refraction testing served for establishing depth to bedrock and the dynamic modulus of elasticity of the rock mass.

The detailed geological and rock mechanics investigation scheme identified three main zones in the foundation for the dam (Fig. 3.2/2). The greater part of the valley floor and the adjacent right-hand flank are made up of little jointed massive granitic gneisses. Attention had to be given in the design and during excavation mainly to the presence of stress-relief joints parallel to the ground surface.

The toe zone of the left rock abutment includes extremely schistose gneisses which tend to contain high amounts of mica at certain locations. The presence of several gouge-filled joints, several centimetres in thickness and parallel to the schistosity planes dipping downslope at some 40 degrees or obliquely towards the upstream, indicates that movement in response to potential tectonic action may here be greater than in the adjoining gneisses.

The left abutment consists of little jointed but bedded gneisses dipping upstream at a medium-stEEP angle. These are uncertain, especially in the centre portion of the left abutment, by granitic gneisses similar to those oc-

Prepared rock foundation surface directly before the construction of the thrust block.
Kölnbrenn Dam

Foundation treatment consisted of a single line of grout holes sunk from the inspection gallery. The foundation rock turned out to be more or less impermeable, so that initial grout take remained relatively small. Contrary to what could have been expected from world-wide experience, however, the schistose locations tend to be very permeable in places. This has been confirmed by observations made during secondary grouting in 1989 (Fig. 3.2/5). The foundation is bounded by stable grouting and regarded as being watertight.

During the first reservoir filling phases, in which the designed maximum operating level was reached twice, excessive stresses developed in the foundation of the dam (Fig. 3.2/5). This led to substantial leakage and high uplift pressures. All the attempts at sealing the leaks by grouting, artificial freezing, or the subsequent provision of an upstream apron intended to relocate the tension zone further upstream yielded no lasting results, so that the decision was finally taken to construct a thrust block against the downstream face of the dam.

The construction of the apron necessitated the provision of two radial tunnels at the end of the apron to connect the existing grout curtain with the new one. The tunnels are large enough for a man to walk in them, and they pass through the dam and continue for some distance into the foundation. Detailed geological documentation of these tunnels shows that the deformation patterns in the granite and in the zones of schistose gneisses are completely different (Fig. 3.2/5). A vertical crack up to 8 m deep in the continuation of the upstream dam face, with lasting openings of up to 20 mm, was characteristic of the schistose zone, whereas in the hard granitic gneisses, the cracks concentrated in the rock contact zone affected by the blasting works for the foundation excavation and to a minor extent in the adjacent concrete. This is an absolutely unprecedented example of the deformation pattern below a highly loaded large arch dam being documented on the prototype.

The foundation of the thrust block is more symmetrical than that of the dam in terms of expected deformation be-
Fig. 3.2/5. Geographical documentation of cut-off trenches at the foundation contacts of Blocks 12 and 20
haviour of the rock. The zone of schistose gneiss is located approximately in the centre of the downstream portion of the arch, where the higher loads occur. Also, in the two abutments massive gneisses are exposed which promise fairly well-balanced foundation reactions (Fig. 3.2/5).

While for the dam foundation treatment by grouting was required only locally, rock consolidation grouting is planned to be carried out along the downstream edge of the thrust block in both abutments. Although the original grout curtain sunk into the valley floor from the inspection gallery was found to be largely intact (Fig. 3.2/6), extensive sealing and consolidation grouting has been performed as part of the repair.

Quarried material has been used as aggregate for both the dam and the thrust block, as natural gravel deposits were not found to be available in sufficient quantities in the vicinity of the dam site. Schistose intercalations rich in mica that were occasionally encountered in the quarry were eliminated.
3.3 Rock Mechanics

Preliminary investigations were undertaken for both the dam (between 1974 and 1977) and the supporting structure for establishing the deformation and strength properties of the rock. This included a great number of in situ tests and in particular laboratory tests.

In situ investigations comprised radial jack tests and seismic refraction tests in both abutments. In the laboratory, rock specimens and drill cores from undisturbed, from disturbed and jointed areas, and from typical crack areas were tested to establish the deformation properties and strengths of the rock mass as a function of direction of loading and direction of schistosity planes (6).

The rock characteristics so determined were used as a basis for determining the rock characteristics to be entered in the static and geometrical analyses with allowance being made for structural features, intensity of jointing and direction of loading from the dam.

Among the rock mechanics studies preparatory to the remedial project for the Kölnbrenn dam were geographical analyses to determine stresses in the foundation rock. Analyses were also made on a two-dimensional model using the method of finite elements. The model simulates the dam at its centre section in the middle of the valley, where the dam is highest, the downstream thrust block including load-transfer structure and an adequate body of rock mass.

Three-dimensional arch action of dam and supporting structure was simulated by means of springs arranged along the centre lines of these structural elements.

The analytical model used is very well suited to demonstrate the effects of the supporting structure. Radial displacements are halved, rotations are reduced to two-thirds of their original magnitudes. This implies a corresponding reduction in normal and shear stresses, and a reduction to zero of tensile stresses from the upstream dam base. Comparison of zones of equal stresses in the foundation rock of the dam shows the amount of stress relief afforded by the prop (Figs. 3.3/1 and 3.3/2).

**Rock Mechanics Characteristics**

<table>
<thead>
<tr>
<th></th>
<th>Elasticity modulus</th>
<th>Compr. strength $c_p$</th>
<th>Cohesion $C$</th>
<th>Angle of friction $\phi$</th>
<th>Shear strength $\tau$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive gneiss</td>
<td>30000 to 35000</td>
<td>90</td>
<td>6.0 (0.8)</td>
<td>1.6 (0.7)</td>
<td></td>
</tr>
<tr>
<td>Bedded gneiss</td>
<td>23000 to 30000</td>
<td>60 to 90</td>
<td>1.3 (0.18)</td>
<td>1.0 (0.7)</td>
<td></td>
</tr>
<tr>
<td>Schistose gneiss</td>
<td>15000 to 17000</td>
<td>30 to 40</td>
<td>1.3 (0.1)</td>
<td>1.1 (0.8)</td>
<td></td>
</tr>
<tr>
<td>Hearting concrete of dam</td>
<td>18000 to 22000</td>
<td>40</td>
<td>4.5</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>Concrete of thrust block</td>
<td>15000 to 20000</td>
<td>25</td>
<td>4.0</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

1) Values in brackets indicate strength after shearing
2) $\sigma_n$ normal stress

---

**Fig. 3.3/1: Results of direct shear tests**

**Drill cores**
3.4 Concrete Technology

The remedial project for the Kölnbrein arch dam relies on mainly two concrete types—the mass concrete for the thrust block supporting the dam and the structural concrete for the load transmission structure.

The compressive strength requirement for the concrete to be used for the thrust block is a 10% percentile of 20 N/mm² at 180 days. While there are no special requirements regarding water tightness and frost resistance, a particularly high degree of crack resistance is considered essential. This implies that concrete development should aim at a material that satisfies the strength requirements while exhibiting favourable placement and compaction properties and low heat generation so as to minimise the stresses due to temperature variations.

Experience gained at previous dam projects and in a comprehensive preliminary exploration programme has given a mix composed of 55 percent Portland cement and 45 percent fly ash as being best suited for this purpose. With an extremely low heat of hydration of 240 Joule (58 cal/g) and a strength at 180 days of 64 N/mm² in accordance with Austrian Standard ÖNORM B 3310 this cement is of particularly high standard strength. The cementing material is produced in the on-site mixing plant with 80 percent by weight of fly ash cement FAZ 30 (30 percent fly ash) and 20 percent Flusol FAZ 30 cement also used for all structural concrete. Flusol is a fly ash ground and supplied by the St. Andra thermal power station and answering the requirements of ÖNORM B 3320.

Aggregate for the concrete to be used for the repair of the Kölnbrein dam is quarried from in situ albite-bearing gneisses with more or less schistose intercalations in the reservoir area. After crushing the aggregate is processed to grain sizes 0/4, 4/12, 12/32 and 32/100 mm. The high mica content of between 1.7 and 2 percent in the 0/4 mm sand and the unfavourable shape of the fragments in the larger particle sizes with a grain index of up to 2.6 lead to an increased tendency to breakage, high water consumption for achieving adequate concrete workability and, hence, a reduced strength. By use of an air entraining and workability agent, it has been possible to reduce total wa-
On an Austrian dam concrete over the past twenty years. The cracking temperature of the same concrete as measured in the temperature-stress testing machine of the Munich university of technology, for the temperature development established on in the prototype, was as low as 6°C. and workability agent resulted in water requirements as high as 225 l per m³ (water-cement ratio, 0.88), which in turn gave a concrete strength at 90 days of only 23 N/mm². As increasing the cement content in excess of 340 kg/m³ would have led to excessive heat generation, the use of workability

<table>
<thead>
<tr>
<th>Concrete composition for 1 m³ of fresh concrete</th>
<th>Hardened concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate for Könlein</td>
<td>0.14% 26% 540 kg</td>
</tr>
<tr>
<td></td>
<td>4/12 13% 270 kg</td>
</tr>
<tr>
<td></td>
<td>12/32 30% 250 kg</td>
</tr>
<tr>
<td></td>
<td>32/100 31% 640 kg</td>
</tr>
<tr>
<td>FAZ 30 cement</td>
<td>140 kg</td>
</tr>
<tr>
<td>Flueat</td>
<td>40 kg</td>
</tr>
<tr>
<td>Total amount of water</td>
<td>138 l</td>
</tr>
<tr>
<td>Air entraining and workability agent (0.25%)</td>
<td>0.45 kg</td>
</tr>
<tr>
<td>Water-cement ratio</td>
<td>0.7</td>
</tr>
<tr>
<td>Unit weight of fresh concrete</td>
<td>2390 kg/m³</td>
</tr>
<tr>
<td>Stump</td>
<td>4 mm</td>
</tr>
<tr>
<td>Air content</td>
<td>3.1%</td>
</tr>
</tbody>
</table>

Testing of the concrete for the supporting structure using a cement mix containing 45 percent of fly ash has given a maximum adiabatic temperature rise of 22°C, which is in fact the lowest temperature rise measured in spite of this very favourable temperature development, additional on-site measures have been required to ensure that the maximum allowable concrete temperature of 27°C in the concrete lifts near the dam base is not exceeded:

- Addition of flake ice to a maximum amount of 55 kg per m³ of concrete, to reduce the temperature of the fresh concrete to between 7 and 9°C.
- Operation of an embedded pipe cooling system in each 3-m lift of concrete.
- Waiting at least 4 days till the next lift is poured.

The pumped concrete of strength class B 300 (90) required for the stub beams for the bearing elements is a type that normally poses no problems. However, with the poor-quality aggregate available at Könlein, preliminary tests using 330 kg of FAZ 30 cement per m³ and an air-entraining agents for reducing the total amount of water addition as well as the effect of Microsilica were tested. Addition of 8 percent Microsilica slurry led to a 25 percent increase in strength but did not accomplish sufficient improvement. Use of 2 percent workability agent led to the required strength of 36 N/mm², but the stability of the concrete mix was so low that production of a concrete of safe quality at the site was not possible. It was a combination of the two additives 8 percent Microsilica + 2 percent workability agent, that led to a pumpable and stable concrete mix with a strength of 40 N/mm² (quality test). The following characteristics were established for the stub beams in the concrete-pouring year 1950:

Although it was necessary to use poor-quality material as aggregate (crushed schistose albite-bearing gneisses), it was possible to satisfy concrete quality requirements by use of optimi-

<table>
<thead>
<tr>
<th>Concrete B 300 (90) for stub beams</th>
<th>Strength at 90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of samples</td>
<td>105</td>
</tr>
<tr>
<td>Mean value</td>
<td>38.4 N/mm²</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>4.6 N/mm²</td>
</tr>
<tr>
<td>5% percentile</td>
<td>30.9 N/mm²</td>
</tr>
</tbody>
</table>
3.5 Licence Procedure

The High Water Right Authority appointed a panel of ten experts in the fields of geology, rock mechanics, concrete technology, dam statics, dam engineering, dam instrumentation, general hydraulic engineering, steel hydraulics engineering, and mechanical engineering to study the remedial project. Starting in 1986, the experts examined the project both for adequacy in terms of engineering standards to be applied in the individual special fields, and for interaction between the envisaged remedial measures.

At a meeting of the Storage Reservoir Commission in the July of 1988, the project was discussed and deemed adequate to its intended purpose and at the beginning of October 1988, approval was given by the Water Right Authority.

A commission appointed from among the staff of the Verband combine, holding company of Österreichische Draukraftwerke, then thoroughly examined the project and confirmed its adequacy. In the February of 1989, the Board of Österreichische Draukraftwerke decided to build the remedial project for the repair of the Kölnbrein dam and let a contract to a number of high-capacity firms, which had qualified for these works in the preceding tendering action.
4.1 Design of Downstream Thrust Block

4.1.1 Rock Excavation
In spite of the low stress level involved, rock excavation was needed to provide some keying for the supporting structure. The vicinity of the large arch dam called for particular care to be exercised in this work. In no case was the dam abutment to be weakened in any appreciable amount. The geometry of the excavation was selected so that bedrock was reached and a satisfactory shape of foundation contact accomplished. Where this was not possible to the necessary extent, consolidation grouting was performed. The total amount of excavation in rock is approximately 50,000 m³.

4.1.2 Thrust Block
The supporting thrust block (Fig. 4.1/1) is an arch-gravity dam with a total volume of 460,000 m³ and a height of 70 m. Its large base width of 65 m has made it appear advisable to provide a longitudinal joint. Radial joints are spaced some 35 m apart. Each contraction joint has been equipped with two independent grouting systems. One of them can serve for several grout injections. Concrete is poured in lifts 3 m high and up to 4,000 m³ in volume. Pipe cooling systems are installed in each lift near the base of the thrust block and in every second lift further up. The portion of thrust block facing the dam is reinforced in the radial direction in three lifts in each block.

Following the usual practice in dam construction, a great number of galleries and chambers have been arranged in the supporting structure to enable joint grouting, measuring and observation operations and the discharge of seepage flows. The bottom discharge pipe has been laid freely in an access gallery through which the bearing elements will be taken to the points of installation. A new gate chamber has been provided at the downstream face of the thrust block to accommodate the hollow-jet valve and for the transport of the bearing elements. About 130,000 m³ of rockfill is placed against the thrust block to make the structure blend with the landscape.

The Kölnbrein reservoir in the midst of the mighty Tauern mountain range

Fig. 4.1/1: Section through dam and thrust block
The bearing forces between dam and thrust block will be transferred by 613 bearing elements arranged in nine rows. Directly at the base, within a region about 9 m wide, the forces will be transferred through a supporting strip provided with elastomer bearings and a grouting joint. With a full reservoir, the load transferred through the bearing elements will amount to 9,500 MN and that transferred through the supporting strip will amount to 1,700 MN, that is 11,200 MN (1.12 million t) in total.

4.1.3 Stub Beams
To accommodate the 613 bearing elements, concrete stub beams are poured against the dam. Concrete stub beams also protrude from the thrust block. It is in the 32-cm gap forming between the pair of stub beams that the bearing elements are installed.

Each row of stub beams is poured against the old dam and made to cantilever from the rising new structure in a single operation across the complete width of each block. Once complete, there will be nine such horizontal rows of stub beams. Vertical spacing between the rows is 4.5 m, except for the two topmost rows which are spaced 4.5 m apart (Fig. 4.1/2).

Before the pouring of the stub beams, the surfaces of the existing dam are prepared carefully by superpressure water jet. 30-mm bolts are installed every 0.5 m along the edges of the roughened areas.

Heavy duty reinforcement is installed in each stub beam to take up splitting tensile stresses. This reinforcement has been designed for four times the loading of each element, that is, 4 x 15,7 MN = 62.8 MN.

Compressive stresses developing in the concrete directly behind a bearing element under a nominal load of 15.7 MN have been calculated to attain a maximum value of approx. 25 N/mm². Due to load dissipation, these compressive stresses decrease to about 3 N/mm² at the contact with the dam. The reinforcement has been subjected to a quality test on a full-scale model (Fig. 4.1/3).

Sinking heat of hydration and subsequent shrinkage will lead to volume reduction in the stub beams. Especially in the stub beams placed against the old dam, volume reduction

---

**Fig. 4.1/3: Section through concrete stub beams**

**Fig. 4.1/4: Plan view of concrete stub beam on dam**
will be restrained, and this condition of restraint will manifest itself as tensile stresses which cause cracking where concrete tensile strengths are exceeded.

Care must primarily be taken to ensure that the restraints do not become large enough to exceed the initial cracking stresses, which are determined by the strength of the concrete. But the cracking does not prejudice in any way the safe transmission of bearing forces.

Special reinforcement designed to limit potential crack openings is provided. PVC tubing is placed to act as planes of weakness at the third points of the stub beams, while the crack control reinforcement continues across these planes.

The edges of the stub beams poured against the dam, which tend to separate from the dam surface, are kept back by connection bolts installed around the circumference of the stub beams (Fig. 4.14).

4.2 Analyses for Dam and Thrust Block

Several methods were used to analyse the combined behaviour of dam and thrust block and to determine stresses and deformations in the individual structural elements and the foundation. These included integration of simplified shell equations, and the trial load and finite element methods. The results obtained from the different procedures show very good agreement and form the basis for the various stability analyses.

4.2.1 Dam

The modulus of deformation for dam and foundation to be used in the analysis were derived from the results of rock mechanics and concrete testing. They are 30 kN/mm² for the massive grout, 20 kN/mm² for the bedded grout and, for the dam concrete, 18 kN/mm² at the base and 22 kN/mm² at the crest.

Based on these assumptions, the dam and foundation were analysed for the "dead weight", "reservoir full" and "temperature" loading cases using the different computation methods. Analytical results were then compared with measured deformations from the year 1983, when top level had been reached and the processes of plastic deformation had largely come to a standstill.

Calculated deformations essentially agree very well with measured deformations, which allows the conclusion that the assumptions used were correct, as illustrated by Fig. 4.21, which is a graph showing displacements of the downstream face of the dam at El. 1720 m a.s.l., that is approximately 15 m above the valley floor.

With a full reservoir, maximum compressive stress in the arch is 9.5 N/mm² and vertical stress in the base of the centre blocks is 7.5 N/mm² compression at the downstream face and up to 2.0 N/mm² tension at the upstream face. With an empty reservoir, vertical tension stresses of about 1.5 N/mm² occur at the downstream face and vertical compression stresses of about 7.0 N/mm² occur at the upstream face.

Due to the very wide and flat valley floor, arch action in the lowest quarter of the arch dam is very small so that substantial shear forces (approx. 60 MN/m) develop at the base of the middle blocks.

Impressive contrast between nature...

... and technology in different views
4.2.2 Combined Behaviour of Dam, Thrust Block and Foundation

The objective is that with a full reservoir, each of the 613 bearing elements is subjected to a nominal loading of 16 MN. Well-designed arrangement of the bearing elements is intended to ensure optimal distribution of the bearing forces. To this end, the individual elements must be loaded at different heads in the reservoir during the filling process.

For calculating the water levels at which the elements should assume load, and for predicting the magnitude of the bearing forces when they develop, it is necessary to establish as exactly as possible the combined action of dam, thrust block and foundation as a basis for the analysis. A three-dimensional finite element model has been developed for this purpose.

As shown on Fig. 4.2/2, this model simulates the two concrete structures including bearing elements and a sufficiently large area of the rock foundation. The total system comprises some 8,200 elements with linear and square displacement formulations and a total of about 32,000 degrees of freedom. The dam has been modelled with four elements across the section at the base and with two elements across the section at the crest. The 613 elements have been simulated by means of 194 bar elements, each of which may assume the “open” or “closed” position. There are several possibilities for the analytical treatment of this special non-linearity. Step-wise simulation of the reservoir filling and reservoir emptying processes would call for a huge computational effort for each change in condition of the elements. In order to avoid this, the overall system has been condensed in a first computation step to the 194 degrees of freedom of the bar elements. This results in a stiffness matrix of 194 by 194, which forms the basis for the computation of the transfer forces.

Fig. 4.2/1: Deformations at El. 1720 with a full reservoir

Fig. 4.2/3: Closure of bearing elements as a function of water head in the reservoir, Centre Block 16

Fig. 4.2/4: Concrete work on the blocks of the thrust block

Analysis of the propped clan for the “dead weight” and “reservoir full” conditions shows that the presence of the thrust block elements plotted against reservoir water level as obtained from the computation. In order to achieve the desired distribution of bearing forces in the reservoir-full condition, the elements of the lowest horizon must assume load with the water surface at El. 1800 m a.s.l. and those of the top horizon must close with the water surface at 1980 m a.s.l.

Concrete work on the blocks of the thrust block.
- substantially reduces displacements and rotations in the base of the centre blocks of the dam,
- almost halves shear forces at the dam base,
- reduces tensile stresses at the upstream dam face to an insignificant level (Fig. 4.2/4).

The loads acting on the thrust block are small, with the vertical stresses being of a magnitude of 3.0 and 4.0 N/mm² and the maximum arch stresses ranging around 2.0 N/mm² (Fig. 4.2/5).

Fig. 4.2/5: Stresses in centre plane, dead-weight plus reservoir-full loading condition

Fig. 4.2/4: Vertical stresses, Block 16, full reservoir
4.2.3 Numerical Earthquake Analysis

In order to determine the additional forces acting on the elements as a result of earthquake effects, the dam was analysed for the reservoir-full condition.

The analytical model was limited to studying symmetrical mode shapes for the consideration of the most unfavourable loading condition in the centre section of the dam. The division into discrete masses as necessary for the finite element approach was checked against frequencies measured on the dam. Methods of analysis used were that of modal superposition and that of direct integration of the equation of motion.

The following is a table comparing frequencies obtained:

<table>
<thead>
<tr>
<th>Measured/calculated natural frequencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>T.W.L. (1690.5m)</td>
</tr>
<tr>
<td>measured</td>
</tr>
<tr>
<td>modal</td>
</tr>
<tr>
<td>numerical</td>
</tr>
</tbody>
</table>

Table: Comparison between measured and calculated natural frequencies.

4.3 Tendering Procedure, Contract Award and Construction Supervision

The execution of a project as large as that for the repair of the Kölnbrein dam implies a great amount of work for preparing the tender documents and letting the contracts for the multitude of different items in the fields of construction as well as mechanical and electrical equipment.

The construction and grouting contracts represent the largest single sum. Based on preliminary information provided in 1988, which gave invited Austrian construction firms the opportunity to prove their qualifications for the construction of large dams, a limited number of firms were invited to submit their tenders as tendering partnerships by November 24, 1988. Competitive tendering also included the drilling and grouting works, for which the firms were directed to invite in their turn tenderers from certain specified subcontractors.

The eight tendering partnerships that formed from among the invited firms submitted their offers by the date set for the opening of the tenders. The tenders were examined and evaluated in the technical and economic respects in accordance with Austrian standard ONORM A 2050. Points to be established were in particular:
- technical and economic efficiency of the tendering partnership,
- equipment and staff planned to be provided,
- compliance with technical specifications and time schedule, as well as adequacy of price.

The best-ranking firms were invited to talks in order to clarify technical, economic, and legal questions and details of the proposed time schedule. The winning bid was that submitted by Arbeitsgemeinschaft Sperrer Kölnbrein (ASK), a joint venture between the firms Stra-
bag, Hofman & Maculan, Polensky & Zöllner, Ilbau, Porr and Mayrader. The grouting contract was passed by the above ASK to Arbeitsgemeinschaft Injektionsarbeiten Kölnbreinsperre (ALK), a joint venture between Insond, Sonderbau and STUAG.

A site management team of 20 persons, consisting of a construction superintendent, engineers and office staff, take care of construction supervision through the owner in the engineering and economic respects and look after the time schedule.

Cost

The total cost of the remedial works for Kölnbrein dam, supported for its greater part by firm offers and based on estimations for the rest, amounts to approximately 1,900 M. Austrian schillings (190 M. US dollars) in September 1980 prices. This total amount is composed as follows:

<table>
<thead>
<tr>
<th></th>
<th>Millions of Austrian schillings</th>
<th>Millions of US dollars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural</td>
<td>1,260</td>
<td>126.0 (66 %)</td>
</tr>
<tr>
<td>Mechanical</td>
<td>235</td>
<td>23.5 (15 %)</td>
</tr>
<tr>
<td>Electrical</td>
<td>56</td>
<td>5.6 (3 %)</td>
</tr>
<tr>
<td>Contingencies (construction management, headquarters, other contingencies)</td>
<td>302</td>
<td>30.2 (16 %)</td>
</tr>
<tr>
<td></td>
<td>1,913</td>
<td>191.3 (100 %)</td>
</tr>
</tbody>
</table>
5. Construction

The contract for the repair of the Kölnbrein dam was let to Arbeitsgemeinschaft Sperrte Kölnbrein at the beginning of February 1989. Arbeitsgemeinschaft Sperrte Kölnbrein is a joint venture between Strabag – Hofman & Maculan – Polensky & Zöllner – Porr – Ilbau – Mayreder.

5.1 Construction Planning

Construction planning had to allow for the following main marginal conditions:

- A rate of concrete placement as high as 200 m³/h, which was necessary to achieve an adequate speed of vertical rise in spite of the extremely large blocks of up to 1200 m³ in maximum surface area and up to 3500 m³ in maximum lift volume.

- The necessity of a tight construction schedule.

- A period as short as 5 months available for installing site facilities ready for concrete placement to start.

- Availability of facilities from the Zillergrund dam construction site.

Allowance for the above marginal conditions finally led to the provision of a site plant of a magnitude similar to that used for the construction of the original dam. In order to save time-consuming earthwork and rock excavation, the facilities were installed approximately at their original locations. In selecting the site facilities, account was taken both of experience from the construction of the Kölnbrein dam and new technologies.

5.2 Site Facilities

5.2.1 Quarrying, Processing and Hauling of Aggregate

The original quarry left from the construction of the dam was worked from its bottom downwards. For this purpose, overburden had to be stripped and roads constructed to provide access to the new benches. Quarrying was performed in four benches 12 m deep each.

Aggregate preparation had to meet the following requirements:

- ensuring an output of at least 500 t/h,
- feeding the largest possible blocks to the first crusher,
- achieving cubical particle shapes to the largest possible extent,
- sufficient sand production,
- treatment of the polluted water.

Fig. 5/1: Layout plan of construction site

LAGEPLAN MIT BAUSTELLEINRICHTUNG FÜR DIE ERKRÜCKUNG DES STUHELZIMBES
A large jaw crusher with a jaw opening of 1.2 m by 1.6 m was selected for the primary crushing stage. Material leaving this stage was then stockpiled before being led to the secondary stage consisting of two cone crushers and four impact mills. A variety of different crusher adjustments as well as material recirculation ensured that the aggregate underwent several crushing processes and that the desired grading was achieved. Aggregate was screened in the screening tower. The 0 to 100-mm crushed material was washed on double-deck screens and separated into the 3 to 12-mm, 12 to 32-mm and 32 to 100-mm fractions. Three discharge belts arranged one on top of the other in a conveyor belt frame carried the washed and screened particles to ground storage areas. The 0 to 3-mm sand was sent to a classifying plant (Eagle) for further treatment. This plant allowed any desired grading curve to be achieved so as to optimise sand quality. Fine particles smaller than 0.06 mm, the greater part of the mica particles and any surplus percentages arising in the grading curve were eliminated.

The sand was dewatered in two dehydrator screws followed by vacuum screens. This ensured the desired humidity reduction to 8 or 10%.

Two water clarifiers were provided to meet the strict requirements regarding waste water treatment. Waste water was first pumped through a cyclone for the elimination of the coarser sand particles. Then the water was conveyed to the clarifiers, where the fines were made to settle out along inclined stacks of plates with the help of flocculation agents. While the thickened sludge was conveyed from the clarifiers to the sludge basin in the quarry, about 90% of the water came out clarified and was pumped back to the screening tower for reuse. The reduction in water consumption involved was another advantage of this installation.

Conveyor belts were provided for transporting the aggregate to the mixing tower. Wheeled loaders placed the material on the belts through feeding hoppers. Optimal computer control was provided for conveyor belt transport and accurate distribution of the different size fractions in the mixing tower.

**Fig. 5/3: Flow chart for gravel preparation**

1. Feed hopper for quarried material
2. Jaw crushers
3. Stockpiling
4. Secondary crushing with 2 cone crushers and 4 impact mills
5. Screening tower
6. Fine sand cyclone
7. Water clarifiers
8. Mixing water tank with recirculating pump
9. Gravel storage, 0-3 mm (136 t/h)
10. Gravel storage, 3-12 mm (91 t/h)
11. Gravel storage, 12-32 mm (138 t/h)
12. Gravel storage, 32-100 mm (135 t/h)
5.2.2 Cement and Fly Ash
The cement had to be treated in three components. Fly-ash cement with 30% fly ash from two different sources as well as pure fly ash were stored in three silos. The fly-ash cements from the two cement mills were fed via computer-controlled rotary valve feeders to a paddle worm conveyor in a ratio of 2 to 1. After mixing, the cement was transported to the mixing-tower silo. In the mixing tower the fly-ash cement and the pure fly ash were stored separately and weighed additively. In the mass concrete, finally, a cement with a 45% fly-ash content was used.

5.2.3 Ice Production Plant
Flake ice had to be added to ensure that the placing temperature of the fresh concrete did not exceed 8 or 10°C even on hot days. An ice production plant with a daily output of 150 t and a silo holding 60 t were provided for this purpose. Through a rake-type agitator in the ice silo, controlled from the mixing-tower computer, the flake ice was conveyed through a discharge screw and a belt conveyor installation to the weighing plant in the mixing tower. Flake ice was added to a maximum amount of 58 kg per cubic metre of concrete depending on the temperature conditions, and was weighed additively with the water.

5.2.4 Concrete Production
The mixing tower was taken over from the Zillergrundl dam site. Equipped with four 4.5-m³ gravity mixers, this installation was abundantly dimensioned for the work to be performed, so that its total output was in no way affected by the decrease in capacity resulting from the additive weighing of the cement and the addition of ice. The fully automatic mixing tower was controlled from a central computer. The amount of water contained in the aggregate was measured by means of neutron soil moisture gauges, and the addition of water and/or ice was automatically adjusted to the total water requirements.

5.2.5 Concrete Transport
Hopper lorries holding 9 m³ of ready-mixed concrete were used for haulage on the concrete quay between the mixing tower and the blinding as well as for the feeding of the crane buckets. The two 260-kN capacity blinding were installed on two parallel tracks on the two valley flanks. The tracks had been laid for the construction of the dam and required only little adaptation, whereas the anchoring of the foundation structures for the head and tail towers were, for safety, completely renewed.

Other concrete haulage alternatives that had naturally been studied were rejected for economic reasons and for saving time. Subsequent checks based on the experience gathered have clearly confirmed the choice made.

5.3 Preparatory Work for the Thrust Block
The foundation area for the supporting structure called for a large amount of preparatory work, which was performed in 1989. In the first place, about 50 000 m³ of rock had to be excavated by blasting. While the foundation for the concrete structure proper starts at a distance of 20 m from the dam, areas in the vicinity of the dam had to be treated by blasting loose rock, and the frost protection concrete structure at the toe of the dam had to be removed. The main difficulties to be met by these operations came from the stiffness of the flanks, small excavation depths in places, and above all from the necessity of repeated reaming for geological reasons.

As it was intended to excavate the whole foundation by blasting, particular care had to be applied because of the nearness of the dam. The permissible velocity was restricted to 15 mm/s, and each blast was monitored by two vibration measuring instruments. Low-vibration blasting was accomplished mainly by restricting the amount of explosive per firing stage. Preliminary work also included the preparation of the areas intended to receive the stub beams. Treatment by 1000-bar high-pressure water jet from a travelling cradle laid bare the granular skeleton of the concrete to ensure adequate adhesion between dam concrete and stub beams.
5.4 Concrete Pouring for the Thrust Block

5.4.1 General
The construction of the supporting structure was by far more complex than is common in normal dam construction, because additional marginal conditions had a substantial bearing on concrete placement operations and on the order in which the blocks were poured. Factors to be allowed for were in particular:
- A high proportion of lifts directly poured on the rock surface.
- The provision of a longitudinal joint dividing the structure into an upper and lower part facing the dam and a downstream part.
- Construction of a 9-m high base strip poured in two sections, leaving a 1.5-m wide gap between the two, which would have to be poured at a later stage.
- Construction of the stub beams. Complex formwork and scaffolding was required for the stub beams both on the dam and on the supporting structure. Allowance had to be made for reinforcement and a great number of embedded parts.
- The lengthening of the drain outlet and of the bottom outlet line through the supporting structure.
- A large proportion of most diverse sorts of inspection galleries and embedded installations.
All these factors had to be taken into account in addition to the marginal conditions normally to be allowed for in dam construction.

5.4.2 Concrete Placement
For the placement of the concrete, a blondin, a crawler loader and a crawler equipped with eight heavy-duty vibrators formed an equipment unit. For the larger lifts of the thrust block, the two available units were used simultaneously.
Quick-transfer climbing formwork which had already proved extremely useful in the construction of the dam was used for the radial and longitudinal joints. For the 9-m high downstream steps, the forms for each lift were anchored on the step below. Formwork was moved by 23-t telescopic mobile cranes lifted onto the blocks by blondins.

5.4.3 Load-transfer System at the Base of the Thrust Block
The 9-m high base structure had to be erected in two parts. A first part was poured against the downstream face of the dam. Concrete was placed by means of skips using a mobile crane. Construction of the second portion filling the gap between the first part and the thrust block could not be started before the thrust block was nearly complete. For this second part, a concrete pump was used to convey the concrete through the inspection galleries. One of the two joints resulting from this system is equipped with neoprene pads for load transfer and the other one with grouting facilities.
5.4.4 Stub Beams for the Load-transfer System

Apart from the substantial machine capacity to be provided for dealing with the large mass concrete requirements, the organisation of the work for the construction of the stub beams on the dam and supporting structure was a decisive factor in planning the work. Before pouring the stub beams, about 9000 grout anchors with a diameter of 30 mm had to be bored and installed on the roughened surfaces of dam concrete. These operations were carried out in advance and independently of the rest of the construction work, from a travelling cradle.

The formwork, reinforcement and concrete operations for the stub beams on the dam were carried out 6 to 9 m above the respective block tops of the supporting structure. Concrete was placed by means of a concrete pump placed on the block directly below. Concrete was fed to the pump by a blondin through a silo. The formwork for the stub beams had to be adjusted to the varying dam inclinations and curvatures. A formwork system with integrated scaffolding was developed which was easy to move and to adjust to the varying conditions. The design of the system also made allowance for the requirements resulting from the high amount of reinforcement and has performed extremely well when applied in practice. The stub beams protruding from the supporting structure were poured in a single operation with the respective lift.

Here, too, formwork and scaffolding were controlling factors.

5.4.5 Concrete Placement Schedule

Following the completion of rock excavation and the installation of the site equipment, concreting was commenced on September 11, 1989. By the onset of winter early in November, 65,000 m³ of concrete had been poured, so that the greater part of the rock surface was covered with concrete by that time.

After the winter, in May 1990, concreting was resumed and continued till the end of October.

The shutdown days between the ten-day working periods were used for carrying out the necessary works on the stub beams and on the base structure as well as for other secondary
works so as to allow straightforward construction of the mass concrete blocks. With a maximum daily output of 4,425 m³ and a maximum monthly output of 85,063 m³, a total amount of 375,000 m³ of concrete was placed in 1990. This left an amount of 32,000 m³ of concrete to be poured in 1991.

5.5 Construction Work in 1991

Following the completion of concrete placement for the thrust block, the following operations remained to be carried out in 1991:
- Concrete backfilling for the load-transfer elements
- Grouting of vertical construction joints
- Remaining works on the supporting structure
- Backfilling of supporting structure on the downstream side
- Dismantling of site equipment

In addition, the steel plates and the neoprene pads will be installed. The sequence of construction works will have to be carefully planned so as to allow for the installation of these elements.

Concrete placement for the supporting structure and the secondary works have largely been completed. Due to the high technical standard of the work and the observance of the construction schedule it has been possible to ensure that the supporting structure will be available to its full extent for propping the dam as planned. In this way, the joint venture entrusted with the construction of the remedial project has made its contribution to the successful rehabilitation of the Kölnbrein dam.

Fig. 5/2: Concrete placement 1989-90-91
General
As the stability interplay between supporting structure and grouting scheme forms the basis for the Konrbrun dam remedial project, grouting assumes here an unusual and actually unprecedented importance. On the one hand, it is necessary to proceed with particular care to avoid adding another factor of instability. On the other hand, the intended purposes of the treatment should be fully accomplished, i.e.:
- largest possible measure of sealing leakages in the foundation contact and foundation rock,
- consolidation of rock areas beneath the upstream apron and the bases of dam and supporting structure,
- repair of cracks in the dam concrete itself, to ensure both watertightness and the safe transfer of loads.

Grouting Project
The grouting project provides for a total amount of about 130,000 metres of grouting, instrument and inspection drillings to be sunk within a period of about three years, and for the injection into these holes of about 1500 t of cement and 200 t of RODUR resin. The reservoir-filling schedule and prevailing climatic conditions confine on-site work to the period from April to September. This implies great variations in plant and staff requirements, which attain maximum levels of 10 drilling rigs, 10 grouting pumps and between 65 and 90 men.

Ideas Underlying the Grouting Project
The ODK grouting project prepared in cooperation with consultant Dr. Lombardi is based on theoretical considerations regarding the application of stable suspensions and — as far as the dam concrete is concerned — combines this approach with experience gained by the RODUR Group, which has a record of successful dam repairs using RODUR resin injections. A novelty in this project idea is the consistent specification of grouting criteria, where mainly the energy supplied in each pass, taken as the product of quantity and pressure, is subject to a variable limitation.

For the first time in the history of grouting, the aspects of flexibility in application, as specified in the description of the rehabilitation project, have here been combined with ideas regarding the stability implications of joint grouting.

AIDEK
The AIDEK automatic grouting data recording scheme allows continuous monitoring of field work. A five-tape recording installation located directly at the pump records time curves for grouting pressure at the drill hole, amount of grout injected, the product of the two as energy quantity, and the two nearest deformation readings. This data is transformed simultaneously and telemetered to the site manager's office, where the individual grouting processes can be observed online on the screen. At the same time all data is both stored at the construction site and modulated transmitted to Klagenfurt so as to be available for periodic interpretation and continuous checking.

Grouting Materials
The properties of the individual grouting materials have been adjusted to their intended purposes:
- For the rock grout consisting of Portland cement 375 in water with plasticising and cohesive additives (water-cement ratio is about 0.7, decantation is less than 5% and filtrate water is less than 10 cm³).
- For the grout curtain area and for the area under the apron, fine-grained cements with Blaine values greater than 8000 cm²/g in suspension, having similar rheological properties, i.e. with Marsh times of about 40 seconds or τ₀ values around 30 dyne/cm².
- For the dam concrete, mainly epoxy resins which, consistent with the principles of the RODUR method, efficiently seal the grouted cracks and joints and ensure the transfer of forces, even against flowing water and high pressure.

Grout Holes
The grout holes are drilled using rotary and percussion methods in about equal amounts, with core drilling prevailing beneath the apron and in the dam concrete. The diameters adopted vary between 46 and 78 mm. Maximum drill hole depths are 60 m as compared with a mean depth of 25 m. Hole spacing is 6 m in a primary grid. Staggered superposition of a secondary grid with the same spacing gives a final average spacing in the plane of the cracks of about 4.7 m. Geologists prepare geotechnical surveys of many of the drill holes, using television cameras.

Checking
The success of the treatment is checked by core drillings, water pressure tests, deformation observation by means of sliding micrometers and extensometers, grout take statistics and the study of pressure, volume, and volume over time versus time. One of the aims of this high amount of time and money spent for checking the grouting operations is to make sure that the purpose of the project is reached and to offer the possibility of adjusting the adopted measures at any time.

Drilling from the inspection gallery
7. Load Transfer System

7.1 Development of the
Load-transfer System and
Qualification Test

Underpinning the slender Kölnbrücke
dam with its relatively large
defonnations by means of the short and
rigid thrust block has to allow for the
differential deformation patterns of the
two structures. The bearing elements
must safely transfer the bearing forces
amounting to a total of 10,000 MN (1
million t) and absorb differential defo-
nations between dam and supporting
structure without developing shear. In
addition, provisions have to be made
for measuring the load transferred.
When the elements are under load, it
must be absolutely impossible for any
outside agent to change the bearing
force which depends on the head of
water in the reservoir. In addition, there
should be a possibility of changing the
distribution of loadings and replacing
or supplementing the load-transfer
elements.

These requirements are met by a modified
design of neoprene pads that have a long record of successful application
in bridge and building construction. For the project under discussion,
these pads are used in combination with a system of adjustable wedges (Fig. 7.1/1).

In order to establish the qualification and good functioning of the individual
structural elements of the load-transfer system, including neoprene pads,
 wedge system, pressure measuring
device and concrete stub beam, a
great number of model tests were performed during the development and
design stage. A full scale model of the
final design now complete has been
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to an extensive testing scheme. The

critical point of these tests was the
application on this model of four times
the nominal load, i.e. 4 times 16,000
kN, which the structure withstood with-
out damage. Following these tests, the
test set-up was subjected to a test of
long duration in order to study its be-
behaviour through an entire reservoir-fil-
ling period. As this comprehensive
testing programme proved both the
structural components and the overall
system to perform as planned, serial
production was started.

Fig. 7.1/1: Section through stub beam with load-transfer element

Wedge system for load transfer
A wedge unit consists of a double wedge screwed to the neoprene pad and resting its weight on brackets projecting from the steel plate on the side of the thrust block; and two adjusting wedges with a taper of 1 to 10. After prestressing, the two adjusting wedges are fixed in the desired position relative to the double wedge by two anchor rods and four adjusting and clamping screws. Prestressing is accomplished by drawing the two adjusting wedges together through the anchor rods by means of two hydraulic cylinders.

Materials containing at least 18% of chromium are required for the wedges, bolts and cladding materials as a precaution against the expected high humidity of the air. The great hardness of between 240 and 300 Brinell hardness of the wedge and cladding materials and the high accuracy requirements make great demands on tools and machinery.

ODK have entrusted Technische Verwaltungs-und Forschungsanstalt (TVFA) at the Vienna technical university with the quality supervision especially of the corrosion-resistant high-alloy steels for wedges, anchor rods and cladded steel plates. The neoprene pads supplied by SHW/RWE, are intended to equalise all the inevitable inaccuracies in parallelism, to absorb differences in transverse displacement from partial to full reservoir loads between dam and thrust block, and to allow a continuous check to be made on the loadings acting on the elements by means of the pressure measuring device arranged at the centre of the bearing and by means of the strain gauges at the outer surfaces. The instruments are also supplied by SHW/RWE.

The reinforced neoprene pads are fabricated following German standard DIN 4141. A large measure of accuracy and control of quality and manufacture is applied to accomplish maximum uniformity in the deformation behaviour of all the bearings. The reinforcing plates with the instrument connections are fabricated by Reisner & Wolff Engineering at Wels, using stress-relieved plates supplied by Voest-Alpine Stahl A.G. The polychloroprene-plastomer mix is fabricated and vulcanised on computer-controlled installations by Metagomma Verona. Each finished bearing is tested on a 40,000-kN testing unit especially designed for this contract by Reisner & Wolff Engineering. The loadings applied are twice the nominal load and six times the transverse displacement, with the pressure and strain gauges being calibrated at the same time. The Profan Anlagenbau von Landausserwegen at the Munich technical university is in charge of the supervision of all quality control measures.

By the execution of this rather unusual contract, the firms of Arge Kraftübertragung are giving another demonstration of their versatility.

7.3 Materials Handling and Hoisting Equipment

The individual parts to be handled and installed weigh up to 2000 kg. The components of the load-transfer system, as steel plates and wedge systems with neoprene pads, are brought through the transport gallery at the base of the supporting structure and loaded on the rack hoist arranged between dam and supporting structure. The rack hoist, supplied by Almak and especially designed to transport these parts, using the same persons, lifts the parts to the bearing levels, where Mannesmann-Demag electrical single-rail trolleys distribute the parts over the entire width of the structure. With the help of this handling equipment, it will be possible to transport and erect within the short period of six months available all the 699 pairs of steel plates and the 813 load-transfer elements.
8. Steel Hydraulics Structures

8.1 Bottom Outlet

During the construction of the thrust block in 1989, the downstream part of the existing bottom outlet was demolished. The outlet pipe was continued through the new structure and a new control valve chamber was provided at its end at the downstream face of the thrust block.

Already in 1984, during the construction of the upstream apron, the bottom-outlet intake structure upstream of the dam had been taken down and the intake cross section of the coarse rack reduced.

Since October 1990, the bottom outlet has been equipped with two butterfly valves (one for maintenance and one for operation), 2000 mm in nominal diameter, followed by a penstock and hollow-jet valve, 1830 mm in nominal diameter, as a control device. Differential movement between dam and supporting structure is absorbed by a flexible pipe forming the transition between the downstream shut-off unit and the penstock.

The 45-m long above-ground penstock rests on slide bearings and is carried by a thrust block transferring forces to the foundation.

Auxiliary equipment is controlled and automatically coordinated by a common pressure oil system (50 bar). Telecontrol of the bottom outlet from the control centre of the Malta development is possible.

The maximum discharge with a full reservoir has been calculated to be 80 m³/s. For compensation water releases, discharge is restricted to 50 m³/s.

8.2 Scour Outlet

The shut-off unit at the downstream end of the scour outlet, an oil-hydraulic driven double-leaf gate 700 by 900 mm in internal dimensions, was relocated several metres nearer to the dam during reconstruction.

Drainage is through an open channel 90 m in length. The emerging jet is averted from the adit located above the outlet.

The intake structure, now closed by the apron of the dam, is planned to be reactivated at a later stage. Maximum discharge will be limited to 12 m³/s (Figs. 8/1, 8/2).

Fig. 8/1: Section through Block 17 of Kölnbremen dam

Fig. 8/2: Kölnbremen reservoir; Discharge and drawdown time for bottom outlet and turbines after extension of bottom outlet and abandonment of scour outlet
9. Instrumentation

9.1 Instruments for the Thrust block

The provision of the downstream prop for the Kohnrein dam has necessitated the enlargement of the existing surveillance system. The new instruments have been designed with special regard for their intended purpose of checking not only the behaviour of the thrust block as an isolated structure, but also the combined action between the two structures and the rock foundation.

Where possible, the instruments have been arranged so as to allow readings to be easily compared with analytical results.

The main instrument planes in the supporting structure coincide with the instrument planes in the dam blocks. The extension of the surveillance system includes the following instruments:

**Pendulums**

To measure horizontal displacement in the radial and tangential directions, one inverted pendulum with a wire length of 95 m has been installed in each one of three blocks of the supporting structure. The point of fixation of the wire is about 45 m below the foundation contact.

**Invar Wire Extensometers**

Invar wire extensometers arranged parallel to the pendulum wires in the pendulum shafts measure vertical movement. The total wire length of 95 m in each block is divided into two equal sections. This permits measuring sections whose readings points are identical with those of the pendulums.

**Rod-type Extensometers**

A total of fourteen extensometers have been provided to measure vertical displacement along the foundation contact. The anchoring points of these extensometers lie about 40 m below the foundation contact.

Radial movement of the longitudinal joint of the supporting structure is measured by three extensometers. The three pendulum stations of the existing dam are connected with those of the supporting structure by means of radial extensometers at the levels of Inspection Galleries 4 and 5 so as to measure differential displacements between the two structures.

40-m-long extensometers starting from the ends of the upper grouting gallery of the supporting structure lead in a tangential direction into the rock abutments.

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Fig. 9.1/1: Section showing instrumentation

Seismic recorder, electronic clock
Fifteen horizontal extensometers distributed over the area of the load-transfer system monitor the radial shortening of the cross section of the stub beams concreted against the downstream face of the dam.

**Sliding Micrometers**
Sliding micrometers, about 40 m in length, have been embedded in the concrete of three blocks of the thrust block to detect potential cracking. Strain in the foundation rock of the supporting structure is measured in the two abutments by two 40-m-long sliding micrometers extending from the upper grouting gallery.

**Clinometers**
Changes in the radial inclination of the supporting structure are measured at all the pendulum stations. Radial tilt of the prop blocks is also measured in the access galleries. In total, twenty-two clinometer stations have been installed.

**Contraction Joint Openings**
The change in opening of all the vertical construction joints is measured at twenty stations arranged in the inspection galleries.
Within the base strip of the supporting structure, where a different load-transfer system has been provided, the movement of the grouting joint is also measured at twenty points in the shafts. Changes in gap opening between dam and supporting structure are measured in three planes at the stub beams in every second row, in three directions each.

**Teleformers and Telepressmeters**
Strains in the supporting structure as well as the distribution of bearing pressure within the concrete structure are monitored by about one hundred electronic sensors embedded at four different levels in the mass concrete of five blocks.

**Concrete Temperature**
A concrete thermometer has been embedded in each pouring lift of two blocks of the thrust block to establish the temperature pattern especially during the setting process. Seventy thermometers have been installed in total.

**Joint and Uplift Pressures**
Joint and uplift pressures in the foundation rock and in the foundation contact of the supporting structure are measured by means of fifteen piezometer stations with pipes extending to a maximum 15 m into the rock.

**Seepage Measuring Stations**
Three V-notch weirs have been installed in the drainage gallery of the supporting structure to measure seepage flow both by sections and in total.

**Geodetic Measuring Equipment**
Horizontal displacements are measured at the level of the downstream berm.
Vertical displacements are measured in the pendulum recesses and also at the level of the downstream berm, in each case on both sides of the vertical construction joints of the thrust block.
About 1200 instrument stations have so far been available for monitoring the performance of the Ktinbrein dam. The provision of the supporting structure has led to the extension of dam instrumentation by the above devices totalling about 320 stations.

**9.2 Measurement of Bearing Forces**
The bearing forces acting on the Ktinbrein dam are measured by pressure measuring devices installed within the neoprene pads. Two methods of measuring bearing loads have been derived from the compression and tension conditions occurring within the reinforced neoprene pads when loaded. These methods have been subjected to a large number of tests on models.

[Fig. 9.2/1: Neoprene pad pressure measurement]
and prototypes and developed to perfection. Each of the 613 neoprene pads of the load-transfer system is equipped with a pressure transducer measuring the pressure within the neoprene (Fig. 9.2/1). This pressure within the neoprene pad as well as the stresses in the reinforcing plates of the neoprene pad (Fig. 9.2/2) exhibit a constant relationship to the load acting on the bearing element. At 47 neoprene pads, the load element is determined from both the pressure transducer and the stress measurement.

This pressure measuring device has been developed in close cooperation between the firms Dr. Brandt/Bochum and Tabal/Vienna.

The pressure measuring device is calibrated in a test stand especially developed for this purpose by neoprene supplier Reiser & Wolf at Wels.

9.3 Data Acquisition and Processing

The following paragraph will deal with the treatment of all metered values, both manually and automatically acquired, their transmission to a computer system, their conversion from "raw" values to results, and finally their storage and representation.

A mobile data collection terminal (MDT) was designed for manual data. The dam supervision staff was equipped with portable data entry terminals with bar code readers. An operator goes from measuring point to measuring point, following the measuring schedule for that particular day. At each measuring point, there is a bar code card about the size of a cheque card showing the designation of that particular instrument also in standard characters. The measuring point is identified by passing the bar code...
reader over this card. For checking, the designation and position of the measuring device as well as the last reading are shown on the display of the portable terminal. The new reading is then manually entered and stored in the portable terminal. At the same time, a marginal check typical of the particular station is made by the software resident in the terminal. After the end of his measuring round the operator re-stores all the values from the portable terminal in a PC accommodated in the supervisor’s room. At present, the data is then transmitted to the Technical Data Processing (TDV) department at Klagenfurt, where it is available for any further interpretations and where it is stored in a specific file.

The mobile data collection terminal went into service in the winter of 1990–91 for gathering 813 instrument readings at the existing dam. With 8970 readings having to be collected in the supporting structure when complete, a total of 9783 readings will then have to be collected manually or automatically.

A new electronic system is planned for monitoring the supporting structure and the nappe pads. This will consist of the following main components:

- **4 Data Loggers** in the supporting structure,
- **the Transmission system,**
- **a Process Control Computer in the Malta Main Station,**
- **the operating controls and output equipment in the dam,** in the supervisor’s room, in the Malta Main Station, and at the company headquarters at Klagenfurt.

The four Data Loggers are accommodated in two special instrument chambers. Communication cables constantly transmit instrument values from the sensor-equipped measuring points to the chambers. There, the values are scanned by a telecontrol system, which digitizes and transmits the data by two physically separate paths (gallary cable, Microwave System) to the power station of the Malta Main Station situated some 20 km from the dam site. The data is then processed by a process control computer (PRA) and stored on files (hourly mean values for 40 days, daily mean values for one year). When the remedial project is complete, the data collected by means of the portable terminals will also be transferred to this process control computer (PRA). There, this data is checked against the automatically collected data. The long-time archive file is stored in the database system of the Technical Data Processing Equipment at the company headquarters. The PRA processes 11,424 data points. Output of dam supervising and performance check data is in the central control station of the Malta Main Station, at the company headquarters and in the Klagenfurter supervising room.

Apart from the usual output equipment, as video monitors and printers, there is a symbol board indicating alarms and the main measured and calculated values. This will be renewed to include additional indications from the supporting structure. In the design of the supervising equipment for the supporting structure, particular emphasis has been placed on protection from atmospheric overvoltages. Appropriate precautions have been taken both in the structure itself, by installing a meshed grounding network, and in the individual systems and their components. All the devices are equipped with uninterruptible power supplies. They have been amply dimensioned to ensure an appropriate level of reliability for the monitoring system. The process control computer was designed so as to be capable also of processing the instrument values from the existing dam after the completion of the remedial project, as the dam’s present supervising system, put into service in 1976, will have to be replaced.

Malta Main Stage and penstock
10. List of Contractors
(with major contract sums)

10.1 Construction, Drilling
and Grouting

Arbeitsgemeinschaft Sperre
Kölnbrein (ASK)
c/o Strabag Österreich AG
Salzburger Straße 323, A-4021 Linz

Joint venture consisting of:
Strabag Österreich AG
Salzburger Straße 323, A-4021 Linz
Hofman & Meculan Bau-AG
Ungargasse 59-61, A-1030 Vienna
Polesky & Zöllner
Bau-Ges.m.b.H.
Julius-Weber-Straße 12,
A-5020 Salzburg
Allgemeine Baugesellschaft –
A. Porr AG
Rennweg 12, A-1031 Vienna
Ilbau Ges.m.b.H.
Ortenburgerstraße 27,
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Mayreder, Koll, List & Co.
Bau-Ges.m.b.H.
Geroldgut 20, A-8010 Graz

Group of subcontractors:
Arbeitsgemeinschaft
Injektionsarbeiten
Kölnbreinsperre (AKK)

Joint venture consisting of:
Insord Ges.m.b.H.
Gloriettengasse 8, A-1130 Vienna
Sondernbau Ges.m.b.H.
Sechshauser Straße 83, A-1150 Vienna
Staag Bau-Aktiengesellschaft
Gutenhofer Straße 19, A-2325 Himberg

10.2 Load-transfer Elements

Arge Kraftübertragung

c/o Waagner-Biro
Stadlauer Straße 54, A-1220 Vienna

Joint venture consisting of:
Waagner-Biro
Stahl- und Maschinenbau
Ges. m. b. H.
Stadlauer Straße 54, A-1221 Vienna
VOEST-Alpine Machinery,
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Engineering Ges.m.b.H.
Postfach 36, Lunzer Straße 78,
A-4031 Linz
J. M. Voith AG Maschinenfabrik
A-3100 St. Pölten
Maschinenfabrik Andritz AG
Stallwegger Straße 18, A-8045 Graz
Arge Elastomerlager,
joint venture between:
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D-7080 Aalen-Wasserfallingen
Reiner & Wolff Engineering
Ges. m. b. H. & Co KG
Oberhart 51, A-4600 Wals
Subcontractors:
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Rombacher Hütte 9,
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TABA Verfahrenstechnische
Geräte GesmbH
Akaziengasse 36, A-1232 Vienna

10.3 Monitoring System

SAT Ges.m.b.H., Ruhmargasse 1-7, A-1210 Vienna

10.4 Cement

Wiedersdorfer und Peggauer
Zementwerke Knoch, Kern & Co.
Ferdinand-Jorgitsch-Straße 15,
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Gebrüder Bernhofer, Zement- und
Kalkwerk, Torren 120,
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